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TITLE: THE SEISMIC CATEGORY I STRUCTURES PROGRAM

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THE SEISMIC CATEGORY I STRUCTURES PROGRAM

by

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ABSTRACT

The Seismic Category I Structures Program entered a new phase at the end of FY 1984. During the prior fiscal years, tests on microconcrete scale model shear deformation dominated structures were completed. The results indicated that these structures responded to seismic excitations with frequencies that were reduced by factors of two or more over those calculated based on an uncracked cross section strength-of-materials approach. This reduction implies that stiffness associated with seismic working loads (loads resulting from an operating basis earthquake up to and including a safe shutdown earthquake) are down by a factor of four or more. These reductions were also consistent with those measured during quasistatic tests to an equivalent level of loading. Furthermore, though the structures themselves were shown to have sufficient reserve margin, the equipment and piping are designed to response spectra that are based on uncracked cross sectional member properties, and these spectra may not be appropriate for actual building responses.

The new phase of the program was aimed at verification of these conclusions using real concrete structures. These test structures were designed based on guidance from the program's Technical Review Group (TRG), a group of nationally recognized nuclear civil structure experts.

During FY 86, a large TRG type structure (4-inch walls of real concrete, No. 3 rebar, and with about 15 tons of added mass) was tested seismically at the construction Engineering Research Laboratory (CERL) in Champaign, IL.

When measured property values were used to predict the first mode frequency as opposed to using assumed design values, results again indicated stiffness reductions on the order of 4, consistent with previous results using microconcrete. Details of the results and floor response spectra obtained from measured results versus floor response spectra obtained from computed results are presented. A plan for further verifying and quantifying stiffness reduction for these structures as a function of shear wall aspect ratio, percentage steel reinforcing and load level is discussed.

INTRODUCTION

The Seismic Category I Structures Program was begun at Los Alamos under the sponsorship of the United States Nuclear Regulatory Commission (USNRC) during the later part of FY 1980. The typical Category I Structure considered is a reinforced concrete structure (exclusive of the containment) that has an aspect ratio such that shear deformation rather than bending deformation governs the response. The primary lateral load carrying element of these structures, the shear wall, is a lightly reinforced wall that is designed to resist missile penetration. Seismic response is normally calculated based on elastic response using an uncracked strength-of-materials approach for calculation of the stiffness contribution to the seismic model. This practice has been followed in the past and specifically, Section 3.1.3.1 of the ASCE's Standard for the Seismic Analysis of Safety-Related Nuclear Structures states that reinforced concrete elements of a Category I structure may be modeled as uncracked sections for the purpose of computing stiffness because the anticipated stress state in the structural elements forming this type of building prior to a seismic event will be in the elastic (uncracked) range.

By the end of FY 84, a number or experimental studies on isolated shear walls, idealized scale model diesel generator buildings, and scale model auxiliary buildings under both quasi-static and seismic loading conditions had been completed and reported (1,2,3). These models were considered scale models based on prototype wall thickness of 30 and 42 inches, but were not intended to serve as scale models of any particular larger structure. Models with 1 inch walls (1/30 and 1/42 scales) and 3 inch walls (1/10 and 1/14) made from microconcrete and simulated rebar were tested. These models were not intended to predict failure loads, crack patterns, etc. for any particular prototype structure. Rather, these structural elements were designed to exhibit under cyclic loading, the nonlinear, softening, hysteretic, stiffness-degradation characteristics that have been observed in other shear wall tests (4,5).

Shortly after the initiation of this program, a Technical Review Group (TRG) consisting of nationally recognized seismic and concrete experts on nuclear civil structures was established to both review the progress and make recommendations regarding the technical directions of the program. The recommendations of this group have been evaluated in light of the needs of the USNRC and, where possible, have been carefully integrated in the program.

This group has reviewed and commented about the program results on items such as aging (cure time), effect of increasing seismic magnitude damping levels, and floor response spectra changes etc. However, the two most commented about results were, (1) the scalability of the tests was illustrated both in the elastic and inelastic range, (2), the so-called "working load" secant stiffness of the models was lower than the computed uncracked cross-sectional values by a factor of about 4.

During their review, the TRG pointed out the following:

1. Design of prototype nuclear plant structures is normally based on an uncracked cross-section strength-of-materials approach which may or may not use a "stiffness reduction factor' for the concrete, but if one is used it is never as large as 4.

2. Although the structures themselves appear to have adequate reserve margin (even if the stiffness is only 25% of the theoretical), any piping and attached equipment will have been designed using inappropriate floor response spectra.

3. Given that a nuclear plant structure designed to have a natural response of about 12 Hz really has a natural frequency of 6 Hz (corresponding to a reduction in stiffness of 4), and allowing further that the natural frequency will decrease because of degrading stiffness, the natural response of the structure will shift well down into the frequency range for which an earthquake's energy content is the largest. This will result in increased amplification in the floor response spectra at lower frequencies, and this fact potentially has significant impact on the equipment and piping design response spectra and their margins of safety.

Note that all three points are related to the difference between measured and calculated stiffnesses of these structures.

To address these stiffness-related issues, it was agreed that credibility experiments should be carried out on a geometry that would appropriately address two important questions. Does our previous experimental data taken on microconcrete models represent behavior that would be observed in prototype structures? What is the appropriate value of the stiffness that should be used in design and for component response spectra computations in these structures?

To address the microconcrete issue, a structure was proposed having the following characteristics:

1. maximum predicted bending and shear mode natural frequency $\leq$ 30 Hz,
2. minimum wall thickness = 4 in. ,
3. height-to-depth ratio of shear wall $\leq$ 1,
4. use actual No. 3 rebar for reinforcing,
5. use standard batch plant concrete,
6. use 0.1 to 1% steel (0.3% each face, each direction ideally),
7. use water blasted construction joints to assure good aggregate interlock.

These structures will be referred to as the TRG structures after the technical review group that established the above criteria.

DESIGN OF THE TRG STRUCTURE AND MODELS

The initial design of the TRG structure was approached from a strength-of-materials point of view. An estimate of the required added mass indicated that on the order of three times the distributed structural mass would be necessary to meet the 30 Hz requirement. It was judged that the effect of the large added mass should be taken into account in arriving at the force-displacement relationship. To support the large added mass, the geometry of Fig. 1 was proposed and designed based on the following analysis. Using the free-body diagram of Fig. 2, and making the usual assumptions regarding bending, transformed sections and effective shear area, an expression for the strain energy for the structure can be written down. Assuming an elastic system, Castigliano's second theorem can be applied to the expression to show that the shear force (V) versus transverse displacement (δ) relationship is

$$\delta = V\left(\frac{hL^2}{2E_cI_t} + \frac{L^3}{3E_cI_t} + \frac{L}{A_eG}\right), \tag{1}$$

where

A_e = transformed section effective shear area,
E_c = Young's modulus for concrete,
G = shear modulus for concrete, and
I_t = transformed section for moment of inertia*, and

the geometric parameters, L, h are defined in Fig. 2. In using the method of transformed sections for these thick wall lightly reinforced structures, we follow standard practice for an uncracked section and include the total concrete area in the calculation not just "compressive" concrete. From Equation 1, we define the stiffness quantities:

$$K_{CB} = K_{\text{Cantilever Bending}} = \frac{3E_cI_t}{L^3} \quad ,$$

$$K_S = K_{\text{Shear}} = \frac{A_eG}{L} \quad ,$$

$$K_{BM} = K_{\text{Bending Moment}} = \frac{2E_cI_t}{hL^2} \quad .$$

The expression for the total stiffness (K_T) then becomes:

$$\delta = V\left(\frac{1}{K_T}\right) = V\left(\frac{1}{K_{BM}} + \frac{1}{K_{CB}} + \frac{1}{K_S}\right) \quad .$$

* For the method of transformed sections as applied to beams of two materials, see Ref. 6, pp. 202-208.

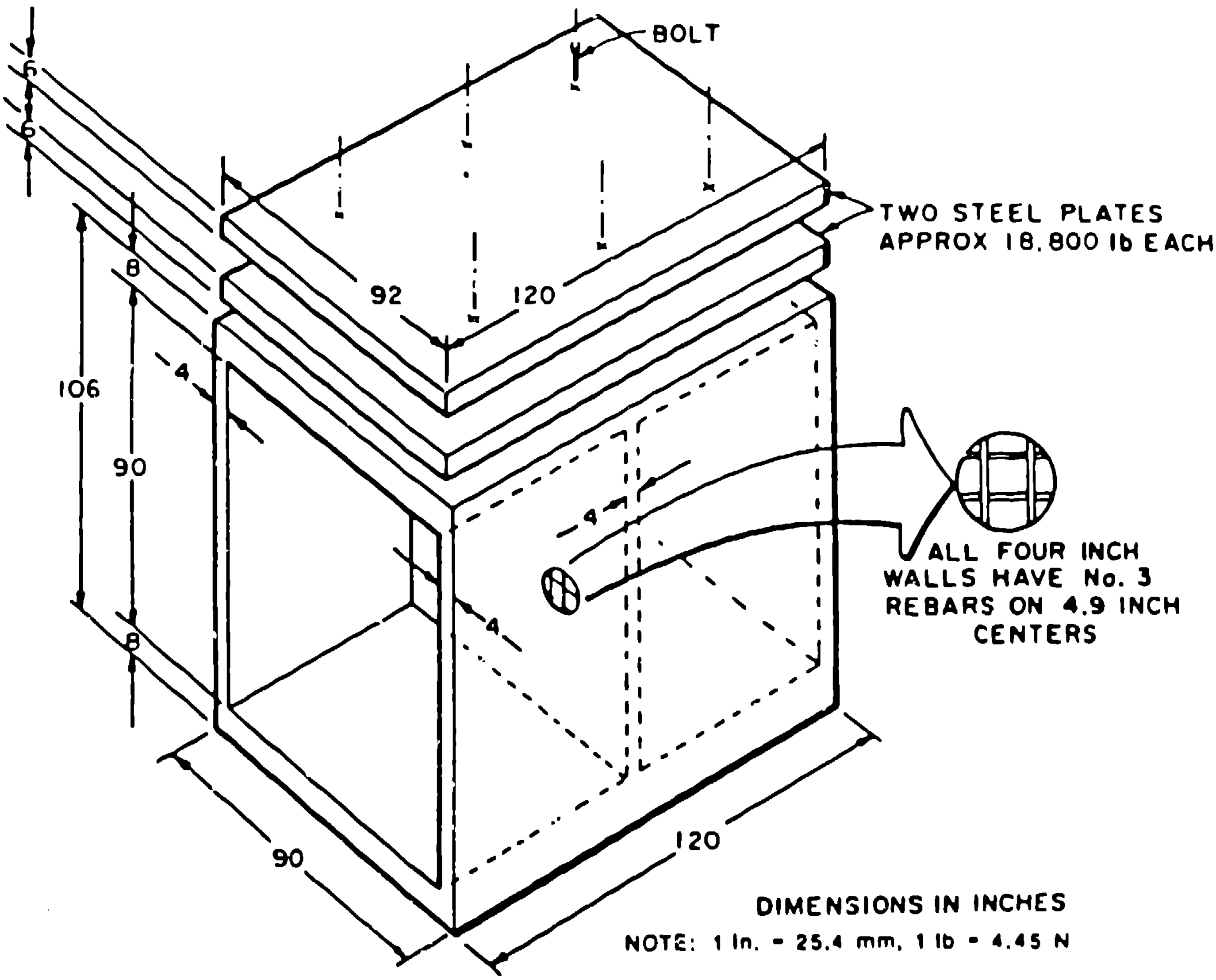

Fig. 1. Geometry of the TRG-3 model. There is a single rebar plane in the center of each 4 inch wall.

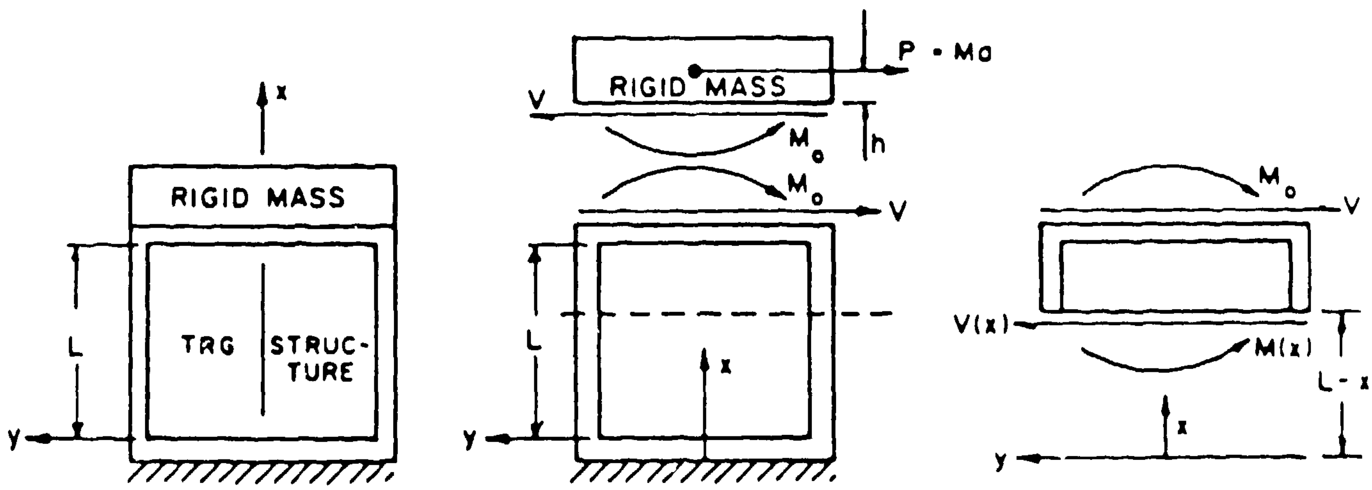

Fig. 2. Free body diagrams used to derive added mass effect on stiffness.

A simple expression for the first mode structural frequency (f) for the bending and shear response then becomes,

$$\omega = 2\pi f = \sqrt{\frac{K_T}{M_A + M_S + M_D}} \quad .$$

where,

M_A = the added mass,
M_S = the mass of the top slab of concrete,
M_D = the distributed mass.

The expression for the added weight required for a targeted frequency can then be approximated as:

$$W_A = 386\left(\frac{K_T}{4\pi^2 f^2} - r\ M_{concrete}\right) \quad .$$

where,

W_A = added weight in lbs,
K_T = total stiffness in lb/in.,
f = targeted natural frequency in Hz, and
$M_{concrete} = M_s + M_D$

The "r" is a multiplicative factor that relates the total structural mass to the effective distributed mass in the vibration mode of interest. Such a factor can be found from a "Rayleigh's Method" analysis of a cantilever beam. In our initial design we used a value for r of 0.3.

Using these equations, the structure of the geometry in Fig. 1 was designed to the recommendations of the TRG. The primary structural element of the TRG structure is a shear wall with an aspect ratio of 1.0. The calculated properties of the structure are given in Table 1. The method of transformed sections for beams of two or more materials was used to compute the geometric properties with a modular ratio of steel to concrete of 10 assumed for this computation and these results are shown in the first column. Because the as-fabricated steel spacing and concrete material properties differed slightly from the design values, column 2 of Table I shows the calculated values of the as-built structure for the geometric properties. Table II shows the various material properties and the basis for their determination that were used in the calculations of Table I.

The structure was constructed from "batch plant" structural grade concrete with a specified minimum compressive strength (f_c') of 3000 psi. Two batch plant truck deliveries containing 3 yards of concrete each were used to facilitate construction. The base and bottom one-foot-heights of the wall sections were placed using the first truck and the remaining portions of the walls and the top slab were completed using the second

TABLE I
COMPUTED CHARACTERISTICS OF THE TRG-3 STRUCTURE

Property	Design Value*	As-Built Value**
Uncracked section moment of inertia(I_t)	= 2.06×10^6 in.^{4}	2.15×10^6 in.^{4}
Area effective shear (transformed)	= 379 in.^{2}	392 in.^{2}
Area (total)	= 1288 in.^{2}	1288 in.^{2}
Total uncracked cantilever bending stiffness ($3E_cI_t/L^3$)	= 2.5×10^7 lb/in.	1.8×10^7 lb/in.
Shear stiffness (A_eG/L)	= 5.3×10^6 lb/in.	3.6×10^6 lb/in.
Mass contribution ($2E_cI_t/hL^2$)	= 2.5×10^8 lb/in.	1.8×10^8 lb/in.
Total stiffness	= 4.3×10^6 lb/in.	3.0×10^6 lb/in.
Max. dead weight normal stress	= 42 psi	----
Max. shear stress in flange at 5 g due to assumed 5% torsion (approx.)	= 35 psi	----
Total concrete	6 cubic yards	----
Total added weight	37600 lb	----
Total weight	61600 lb	----

* calculated using $E_c = 3.0 \times 10^6$ lb/in.^{2} as the design value.

** calculated using $E_c = 2.0 \times 10^6$ lb/in.^{2} from the strain gage measured value.

TABLE II
MATERIAL PROPERTIES FOR TRG-3

Concrete:

E_c	= assumed for design purposes = 3×10^6 psi
E_c	= (measured at σ - ϵ origin) = 2.0×10^6 psi
f_c'	= (compressive strength) = 3807 psi
f_t	= (split tensile test strength) = 351 psi
E_c	= $57000\sqrt{f_c'} = 3.5 \times 10^6$ psi

Steel - Standard No. 3 Rebar*

0.6% Both Directions

E	= 30×10^6 psi
Yield Strength	= 40 ksi min.
Ultimate Strength	= 70 ksi min.
Elongation at failure	= 11% min.
Diameter	= 3/8 in.

* These are handbook values and have not as yet been measured.

truck. The entire concrete placement was completed within a four hour period using concrete vibrators and standard construction methods. At the same time standard 6 in.-diameter x 12 in.-high samples of the concrete were obtained from each truck, in accordance with ASTM C31-83 and C172-82 (Ref. 7, 8) specifications. Standard slump specimens were also obtained from each batch according to ASTM C143-78 (Ref. 9) to ensure proper water to cement ratios.

From Table II, the value of the Young's modulus for concrete is seen to vary from 2 x 10^6 psi for a strain gage measured value to 3.5 x 10^6 psi for a value based on $57{,}000\sqrt{f_c'}$. The reason for this difference is not clear and further samples taken from the structures itself (core samples) will be tested in FY 87 to verify the strain gage determined value. The results computed in the remainder of this paper will be stated using all 3 values of E_c.

TESTING HISTORY OF TRG-3 IN BRIEF

The low load level testing history for the structure will be summarized. The structure was placed on foam pads for modal testing as a "free-free" structure to characterize the very low level vibrational frequencies and thus the structural "as-built" stiffnesses. First, a series of hammer tap tests was used to excite the structure. Second, a 300 lb. portable shaker was used to excite the structure with a random signal having frequency content of 0-500 Hz. For both of these modal analysis tests, accelerometer data was taken at 31 points shown schematically in Fig. 3 on the structure in three orthogonal directions. These tests gave some natural frequency and mode shape information, but the foam pads did not allow a true "free-free" condition to be simulated and coherence for the test signals below 200 Hz was poor. Next, the base of the structure was bolted to a load frame specifically constructed for low load level static testing and states testing was done keeping the load below a value that would produce a calculated maximum principal stress of 40 psi. The 37,600 lbs. of added weights arrived after these tests and were fitted to the structure and the transfer functions of the top slab acceleration to the base slab acceleration records were measured. The structure was then shipped to the Construction Engineering Research Laboratory (CERL) at Champaign, IL.

The structure was suspended from the CERL crane using nylon straps and "free-free" modal testing was again carried out using a portable shaker and random force excitation. In these tests coherence at lower frequencies was good and the modal analysis gave satisfactory results. The structure was next bolted to the CERL shaker table and a seismic test plan was carried out using a modified (properly time-scaled for a 1/5 scale structure and baseline corrected) El-Centro earthquake signal as the input time history.

LOW LOAD LEVEL TESTING RESULTS

In these tests, the initial as-built stiffness of the structure was obtained to compare with theory.

The static load displacement curve was obtained by monotonically loading the structure to 10,000 lb. The results are shown in Fig. 4. A least squares fit of the data was performed (solid line) and the

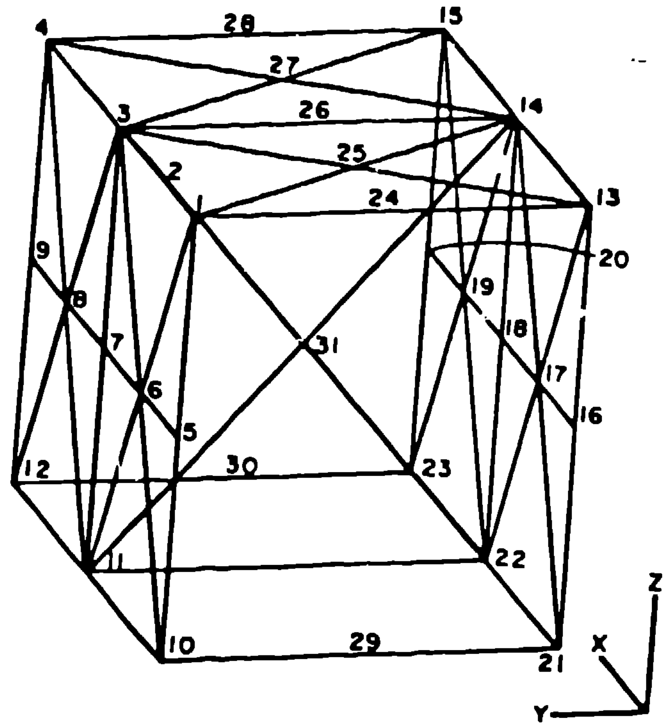

Fig. 3. Location of the accelerometers for the modal test data.

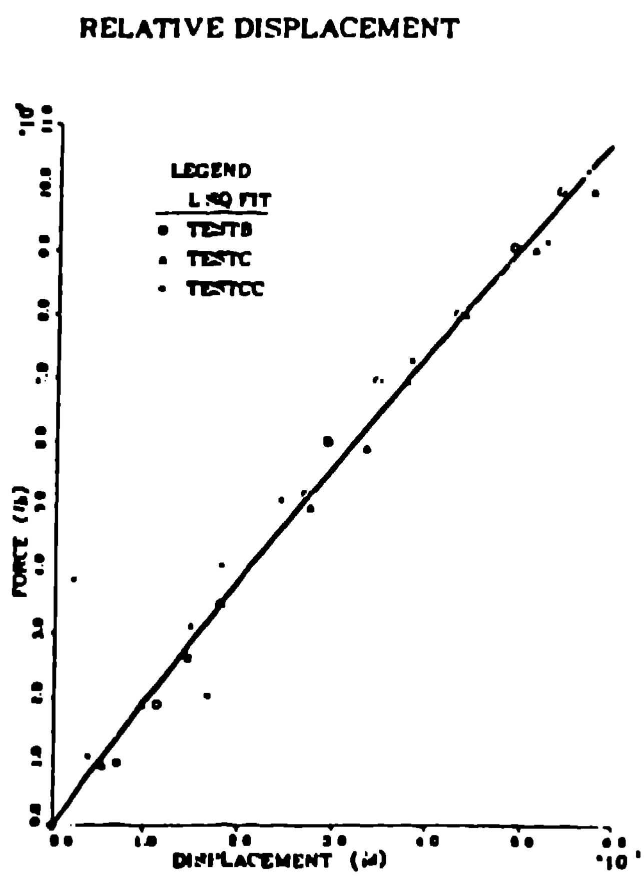

Fig. 4. Static load-displacement result 10,000 lb corresponds to about 40 psi maximum tensive stress in the concrete.

stiffness was evaluated as the slope of this curve at the origin. This value is 1.95 x 10^6 lb/in., which is about 70% of the theoretical value based on an uncracked cross-section and using the strength-of-materials approach. This value is probably low as the connection effects at the base are unaccounted for.

The low load level modal test results gave similar information. The first two modes are the torsion mode and the free-free shear/bending mode. Table III shows the comparison of the measured frequencies.

TABLE III
MODAL TEST RESULTS

	Frequency (Hz)	
	Theoretical	Measured
Mode 1 (Torsion)		
E_c = 2 x 10^6 psi	21	29
E_c = 3 x 10^6 psi	26	29
E_c = 3.5 x 10^6 psi	28	29
Mode 2 (Shear/Bending)		
E_c = 2 x 10^6 psi	87	75
E_c = 3 x 10^6 psi	106	75
E_c = 3.5 x 10^6 psi	115	75

Using the strain gage measured value of Young's modulus for the concrete and the static low load level test data and these modal tests results, it was concluded that the TKG-3 structure at the time of seismic testing had a stiffness value in the shear/bending mode of about 70-80% of the theoretical uncracked cross-sectional value.

INITIAL HAVERSINE AND SEISMIC TEST RESULTS - WORKING LOAD STIFFNESSES

The working load stiffness is defined here as the equivalent linear stiffness that the structure has, as deduced from its response to applying any significant dynamic loading to the structure. In the seismic tests, the lowest load level that can be maintained with good shaker control for this structure is about 0 5 g which corresponds to about 0.1 g on a real Category I structure. This value is used here as the "working load" and corresponds to a typical operating basis earthquake (OBE) loading on a real structure.

A 0.5 g haversine base input pulse was designed to characterize the structure initially and to characterize damage between seismic tests on the CERL table. This type of test is assumed to cause less further damage to the structure than the 1/2 g broad band random type of characterization signal used between seismic tests on our previous models. Figure 5 shows the horizontal acceleration time history at the base of the structure and the corresponding horizontal top slab response in the plane of the shear wall. This record is from the first pulse applied to the structure by the CERL table. Figure 6 shows the real and imaginary parts of the transfer function of this top slab acceleration to the base slab. Both records indicate a clear 10 Hz natural frequency for the shear-bending mode for this structure.

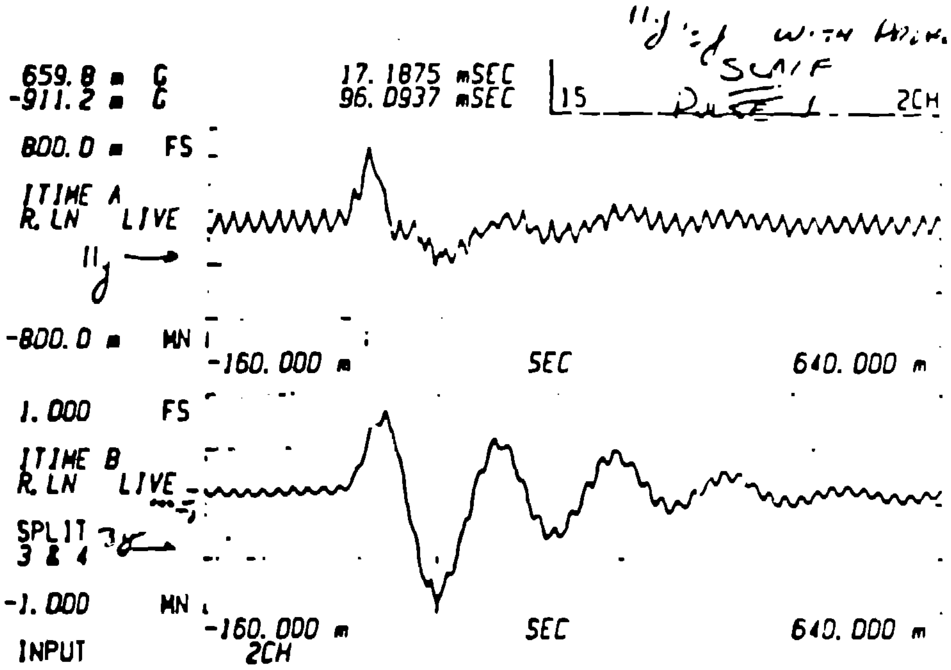

Fig. 5. Time history of the first base input haversine pulse (top trace) and the top response accelerometer. This record was taken "on-line" at CERL.

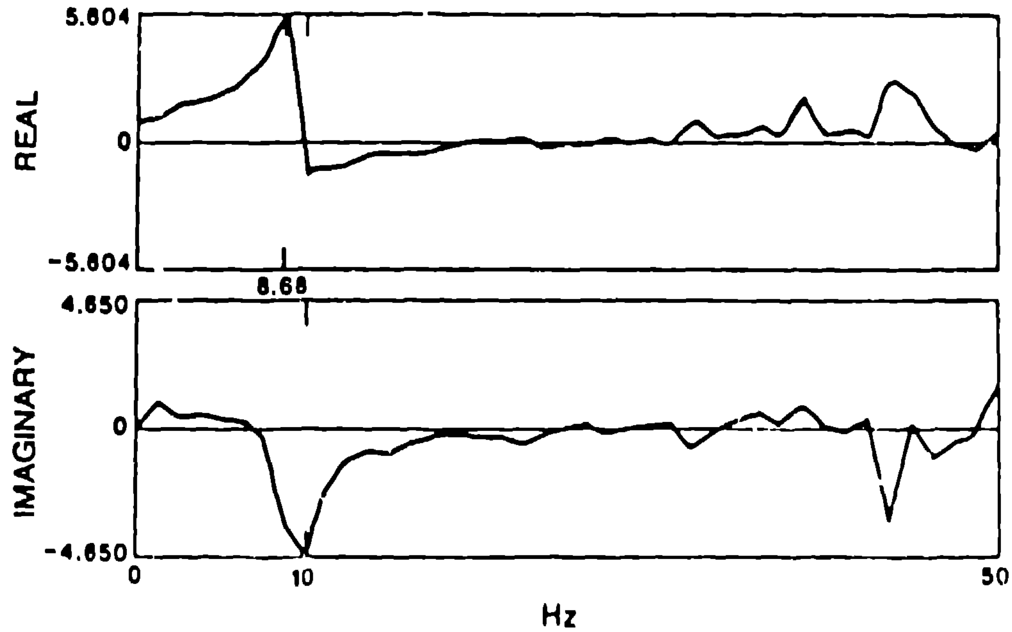

Fig. 6. Real and imaginary parts of the transfer function of the top accelerometer record to the base record for the haversine pulse of Fig. 5.

Figure 7 shows the time history of the base input 0.5 g nominal max peak seismic signal and Fig. 8 shows the response time history. As can be seen from Fig. 7, the base input magnitude was actually about 0.73 g max peak. Figure 9 shows the acceleration transfer function of the top to base for these two signals. The structure predominantly responds to the earthquake in the 10 Hz shear-bending mode.

The working load stiffness implied by this result is estimated by using the relationship that

$$\frac{f_{measured}}{f_{theoretical}} = \sqrt{\frac{k_{actual}}{K_{theoretical}}}$$

Henceforth, we will use K_{actual} and $K_{measured}$ interchangeably, but natural frequency was the actual measured quantity.

The implications of these results with regard to measured versus theoretical stiffnesses will now be discussed. Three methods of approach to the design of the TRG-3 structure will be illustrated in order to quantify the magnitude of the various assumptions that are or have been used in the design of Category I structures.

The first method will be called the "design method". The assumptions for this method are as follows:

(1) Assume an uncracked concrete cross-section;
(2) Use the method of transformed sections to transform steel area to concrete and compute the transformed bending area moment of inertia for the cross-section,
(3) Use the strength-of-materials approach to compute the stiffness (i.e. Eq. 1),
(4) Assume the top and bottom concrete slabs are rigid compared to the cantilever cross-section and compute the effective MASS = $M_{ADDED} + M_{SLAB} + M_{DISTRIBUTED}$.
(5) Assume that the base is fixed.

As previously stated, in order to further quantify the magnitude of various effects, we will give results using the three different values of concrete modulus discussed previously. The ACI method, the design assumption, and the strain gage value. These values are:

(1) ACI Method $E_c = 57000\sqrt{f'_c} = 57000\sqrt{3607}$
$E_c = 3.5 \times 10^6$ lb/in.^{2} ,
(2) Design assumption $E_c = 3 \times 10^6$ lb/in.^{2} ,
(3) Strain gage value $E_c = 2 \times 10^6$ lb/in.^{2} .

Table IV shows the results of calculating the stiffness and natural frequency of this structure based on these assumptions. Clearly the stiffness of this structure under working/loads is lower than theory would predict. Furthermore, it appears that the stiffness decreased in going from a low load level to a working load level.

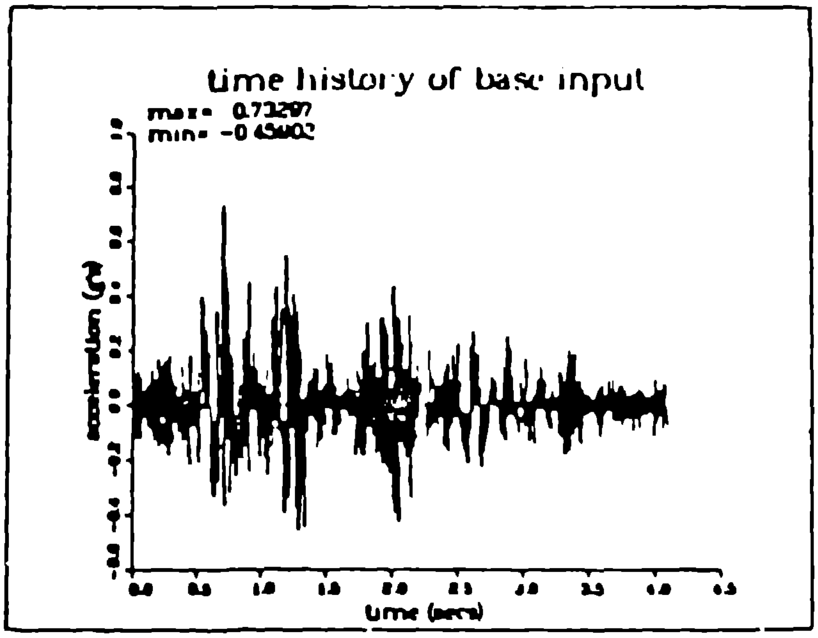

Fig. 7. Time history of the base input for the first seismic test of TRG-3 at CERL.

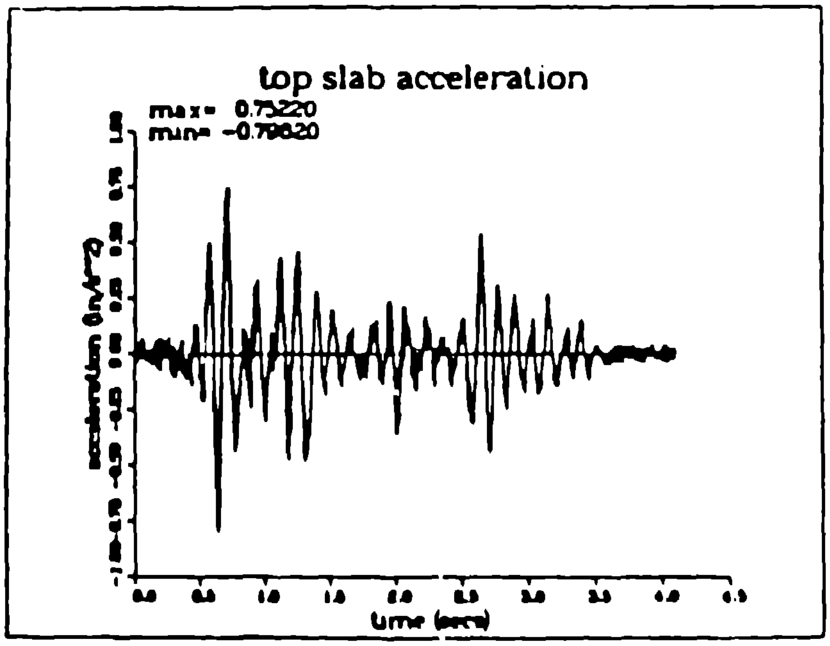

Fig. 8. Response acceleration of the top slab for the first seismic test of TRG-3 at CERL.

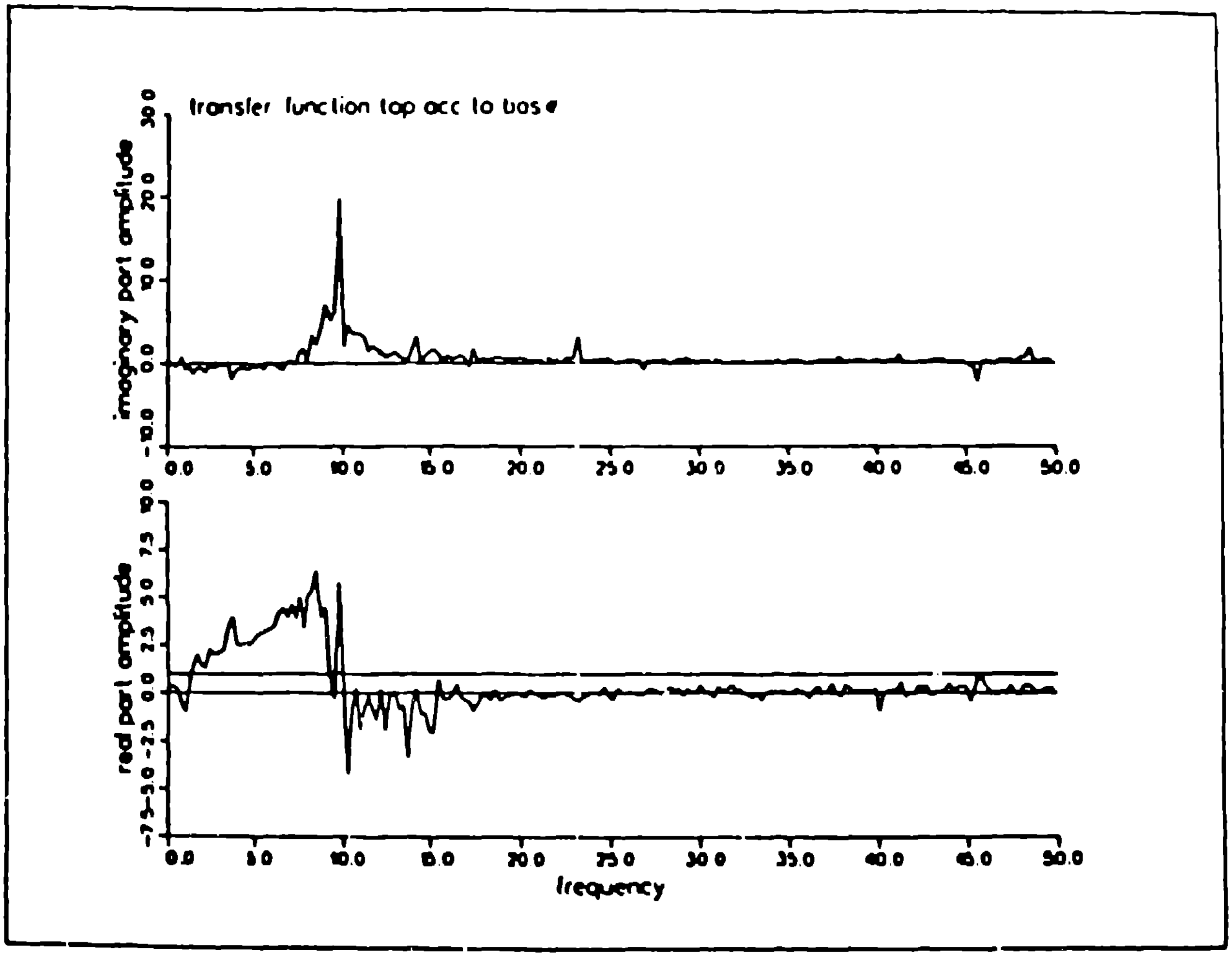

Fig. 9. Acceleration transfer function for the top slab to the base for the test of Figs. 12 and 13. The 10 Hz natural mode is clearly shown.

TABLE IV
SUMMARY OF RESULTS FOR WORKING LOAD
STIFFNESSES BASED ON THE DESIGN METHOD

E_c lb/in.^{2}	Computed Stiffness (lb/in.)	$K^*_{STRUCTURAL}$ / K_{THEORY}	First Mode Frequency under E.Q. conditions: Predicted (Hz)	$f^{**}_{MEASURED}$ / $f_{PREDICTED}$
3.5×10^6	5.0×10^6	0.15	31.8	0.31
3.0×10^6	4.3×10^6	0.18	29.5	0.34
2.0×10^6	3.0×10^6	0.25	24.6	0.40

* $K_{STRUCTURAL}$ = 759,000 lb/in.

** $f_{MEASURED}$ = 10 Hz.

How much load is necessary, and the physical mechanism for stiffness changes has not been completely determined. All evidence points strongly to the fact that the structure does not respond to significant dynamic loads (i.e. working loads) as if it had uncracked shear walls.

The results presented in Table IV are representative of methods that were used by the architectural/engineering firms for existing nuclear plant structures of this type.

To further illustrate the magnitude of various effects an alternative analysis can be carried out. The engineering mechanics specialist might approach this problem from an energy methods point of view and use Hamilton's principle and shape functions to obtain the best single degree-of-freedom representation possible for the TRG-3 structure and its base connections. We will not go through all details in this paper, but the interested reader can obtain the theory from Ref. 10. The details are available from the author. The results for the TRG-3 structure are presented in Table V.

Finally Table VI illustrates modern analysis methods (the finite element method) that might be used on current and future nuclear plant structural designs. Both the fixed based assumptions and an attempt at modeling the connection effects are illustrated here for the three values of the concrete modulus. The ABAQUS finite element code was used with shell elements representing the structure and the smeared rebar option combined with the concrete material model to represent the material. The calculations are totally elastic. The structure was represented using the quarter model mesh shown in Fig. 10 with the appropriate symmetry boundary conditions for the vibration modes of interest. Table VI gives the results for the shear-bending mode.

The calculational basis for Tables IV - VI were chosen to represent three types of design methods of nuclear plant structures. The first basis, the design method, represents the method that was probably used for existing plants. The second basis, the engineering mechanics/ structural dynamics basis, represents the best design method that could

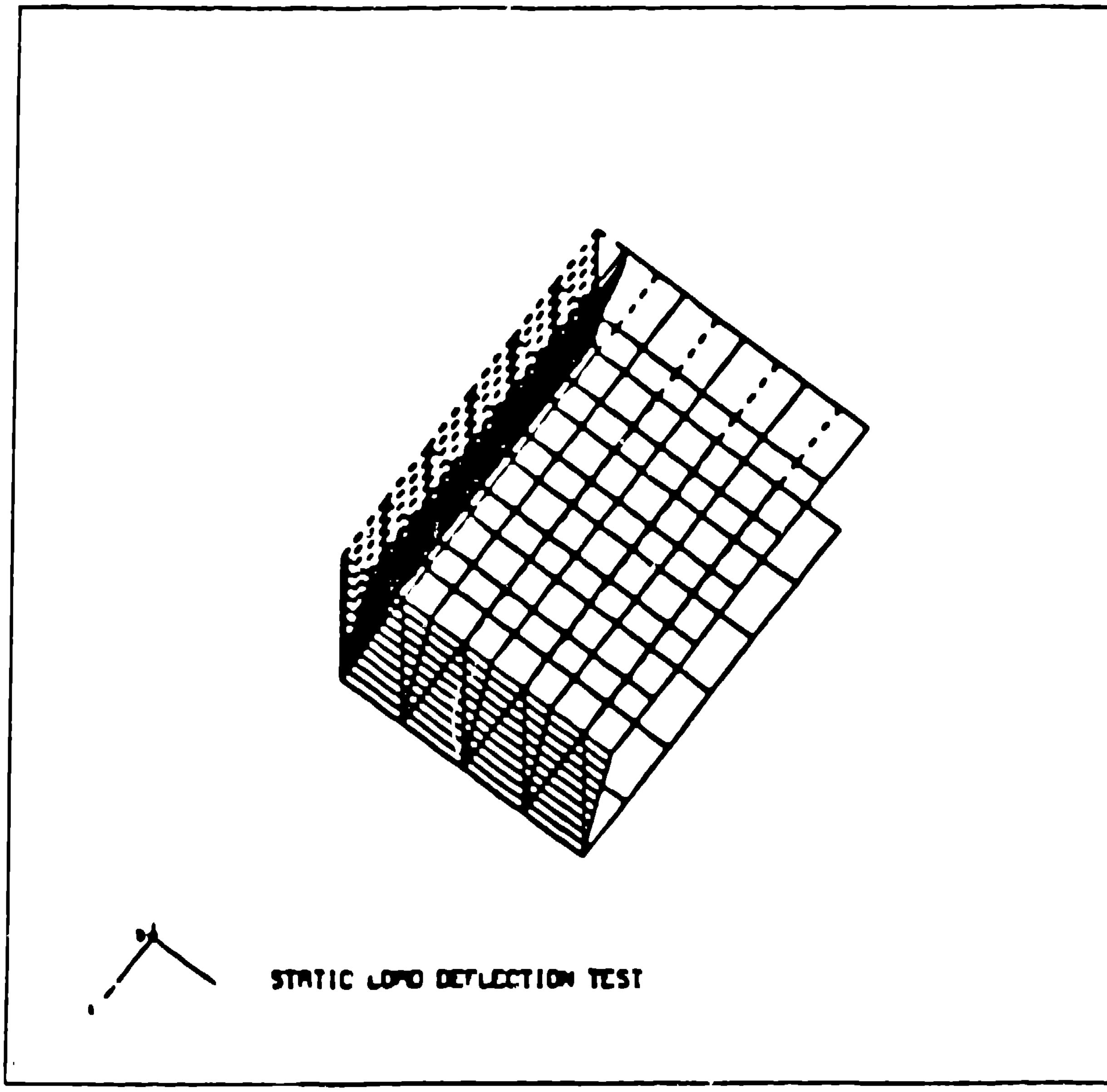

Fig. 10. One-quarter model finite element mesh used for finite element calculations.

TABLE V
ENGINEERING MECHANICS/STRUCTURAL DYNAMIC
"BEST EFFORT" SUMMARY

Computed E_c (lb/in.^{2})	Stiffness (lb/in.)	$K^*_{STRUCTURAL}$ / K_{THEORY}	First Mode Frequency under E.Q. conditions: Predicted (Hz)	$f^{**}_{MEASURED}$ / $f_{PREDICTED}$
3.5×10^6	2.76×10^6	0.23	18.9	0.53
3.0×10^6	2.59×10^6	0.25	18.8	0.53
2.0×10^6	2.11×10^6	0.31	18.0	0.56

* $K_{STRUCTURAL}$ = 647,000 lb/in.

** $f_{MEASURED}$ = 10 Hz.

TABLE VI
SUMMARY OF RESULTS FROM A FINITE ELEMENT MODEL CALCULATIONS

E_c (lb/in.^{2})	Calculated Stiffness (lb/in.)	$K^*_{STRUCTURAL}$ / K_{THEORY}	First Mode Natural Frequency under E.Q. conditions: Predicted (Hz)	$f^{**}_{MEASURED}$ / $f_{PREDICTED}$
Fixed-Based Model:				
3.5×10^6	4.04×10^6	0.16	29.0	0.34
3.0×10^6	3.47×10^6	0.19	26.8	0.37
2.0×10^6	2.33×10^6	0.28	21.9	0.46
Bolts Modeled with Axial Springs:				
3.5×10^6	2.71×10^6	0.24	22.7	0.44
3.0×10^6	2.38×10^6	0.27	21.2	0.47
2.0×10^6	1.68×10^6	0.38	17.9	0.56

* $K_{STRUCTURAL}$ = 647,000 lb/in.

** $f_{MEASURED}$ = 10 Hz.

methodology for "plant structures of the future". Study of these tables indicate that all calculational bases produce results that are consistent with one another and are probably within the variations that would be handled by NRC Regulatory Guides that cover peak broadening. The difference between Method 1 and 2, for example, might represent accounting for soil-structure interaction or not. The point is that the calculational method will not account for the stiffness reduction at working loads that has been consistently measured in this program. Figure 11 illustrates this consistency using the "design method" basis of calculation and strain gage determined concrete material properties.

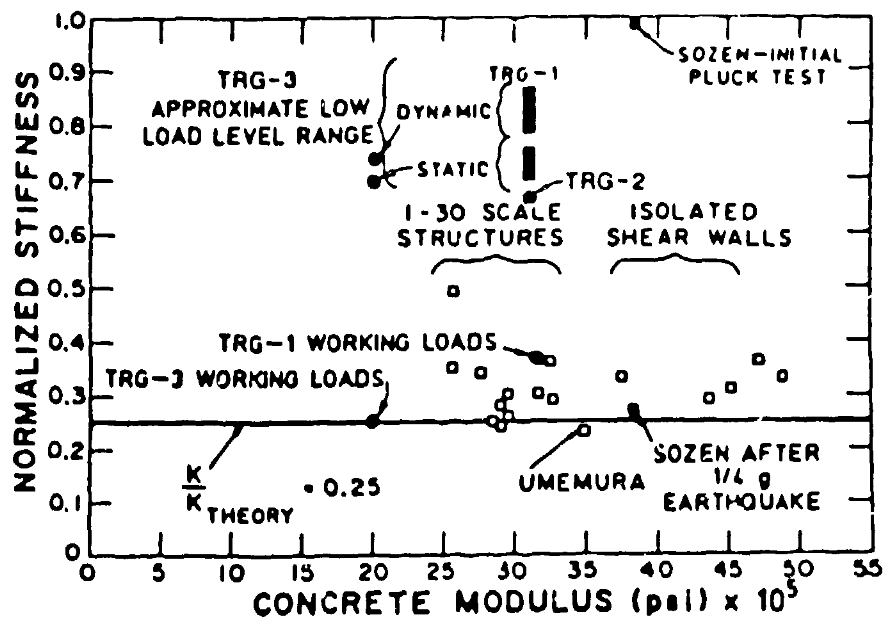

Fig. 11. Normalized stiffness for various tests at Los Alamos and elsewhere.

IMPLICATIONS WITH REGARD TO EQUIPMENT AND PIPING

To illustrate the implications of the "reduced working load stiffness," Figures 12 and 13 have been prepared using the first two calculational bases as discussed, and the response data taken from the top slab accelerometer for TRG-3. These figures compare the design floor response spectra for this particular aspect ratio shear wall structure.

The meaning of this result for existing and future Category I structures is under serious study by NRC, Los Alamos and industry. The ACI 349 code committee has been made aware the result as well as the ASCE Nuclear structures committee and its working groups.

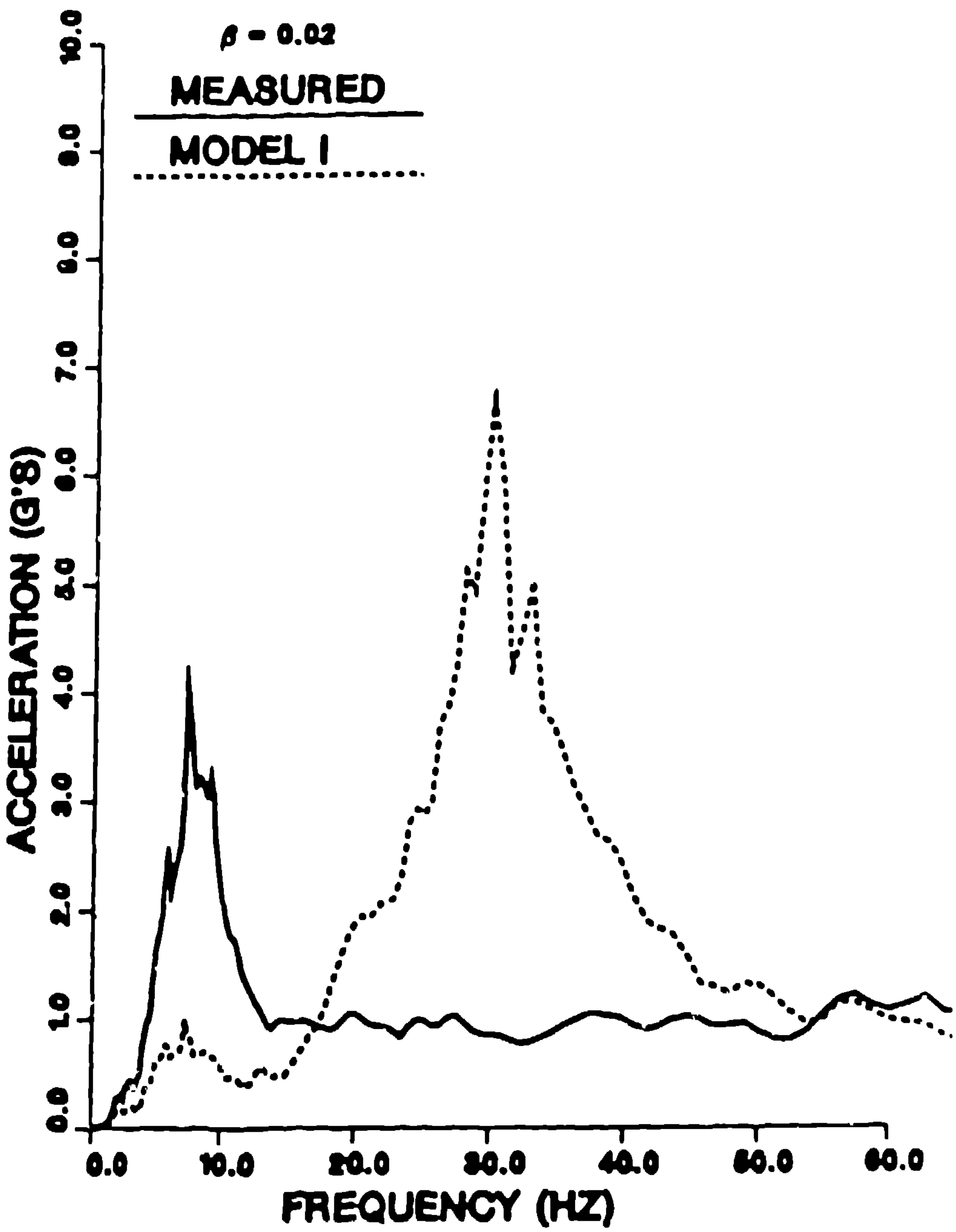

Fig. 12. The floor response spectra as calculated from the acceleration record using the "Design Basis" model compared with the floor response spectra calculated from the top slab accelerometer record.

Clearly further testing is necessary on real structures to verify this result. The TRG has recommended varying both aspect ratio and percentage reinforcing in these models in carrying out such verification. They have also recommended returning to a quasistatic cyclic loading. A set of experiments was laid out using statistical experimental design methods that will begin during FY 87. The end product of this effort should be a model that will predict the stiffness as a function of aspect ratio, percentage reinforcing and load level. The program will continue to work closely with NRC, the TRG, and code committees to make the best use of these research results.

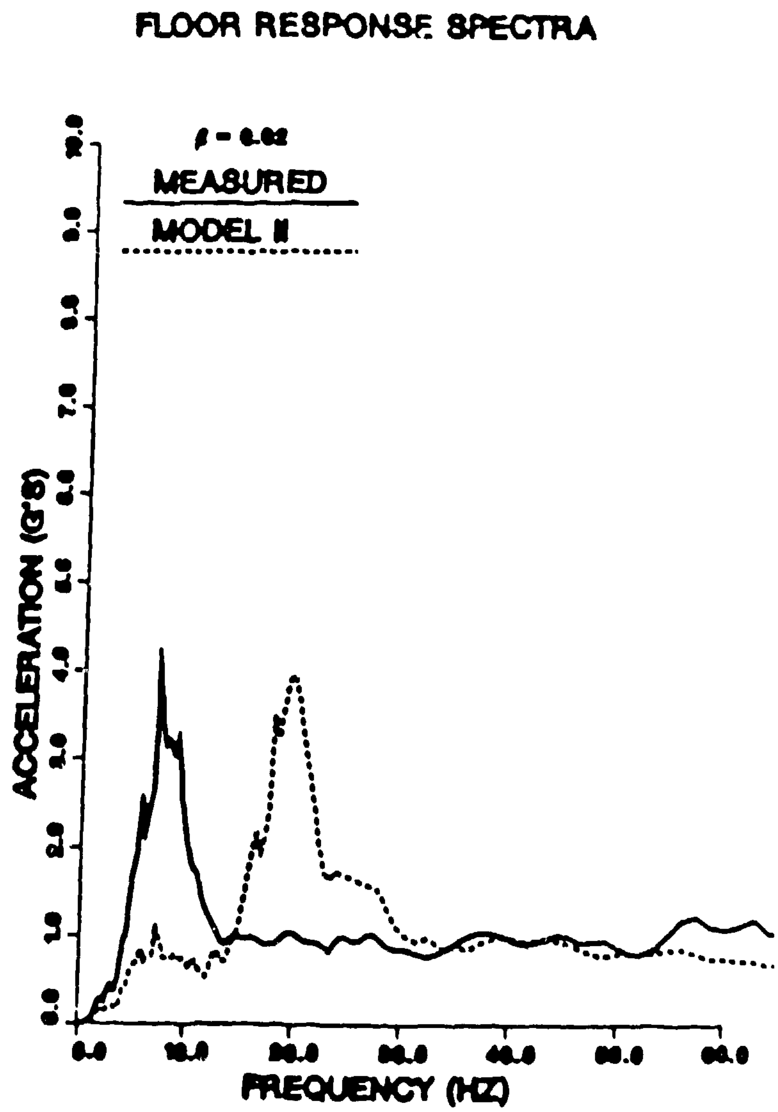

Fig. 13. The floor response spectra calculated from the "Engineering Mechanics" model of the structure compared with that calculated from the top slab accelerometer record.

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